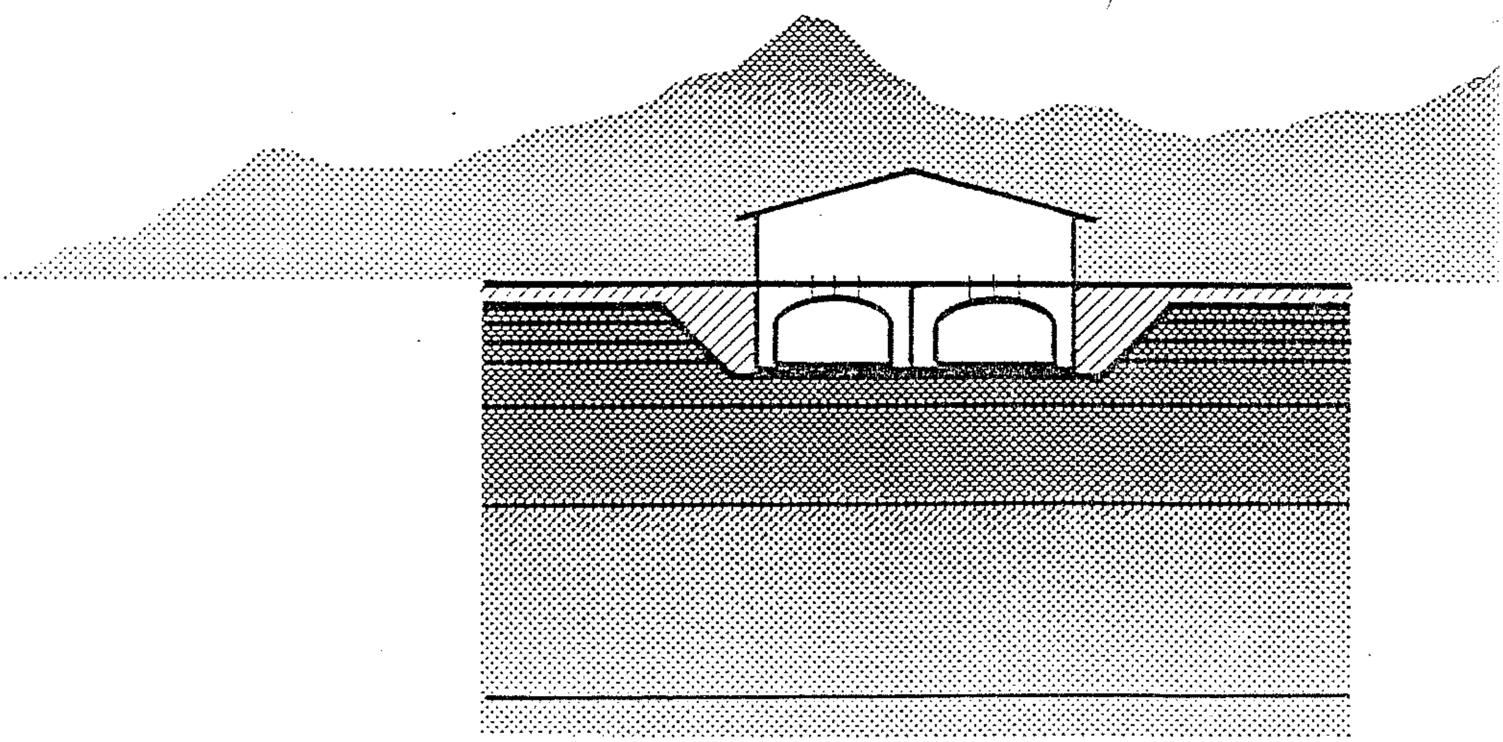
HIGH LEVEL WASTE
TANK FARM
REPLACEMENT PROJECT

T Borschel



Soil Properties Report

**Westinghouse Idaho
Nuclear Company, Inc.**

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CONTENTS

1. INTRODUCTION

This report documents expected soil material properties for use in the design of the High Level Waste Tank Farm Replacement (HLWTFR) Project. It includes a description of available information, assumptions, estimates of expected values, and confidence levels.

The scope of work for this effort is defined in the *Scope of Work Statement* for "Evaluation of ICPP Soils Reports to provide data for HLWTFR Design," Revision 0, October 14, 1991, prepared by Westinghouse Idaho Nuclear Company, Inc.

ICF KE has reviewed available soils investigation reports prepared to date to support the design and construction of facilities in the proximity of the HLWTFR Project site. Based on the available information, this report provides best estimate values for soil materials properties required for the design of the tank vault, as well as a technical basis for the selection of these estimated values, and a description of the confidence level associated with each estimated value. The technical basis addresses such pertinent factors as the distance of each neighboring facility report's boring locations from the HLWTFR Project, methodology used, and consistency of results among the reports.

2. AVAILABLE INFORMATION

Six soil investigation reports which support design and construction of facilities in the proximity of the HLWTFR Project are available to date. The reports are identified in the John W. Viita letter to Ali Jaafar of WINCO on the subject of HLWTFR Project soils investigations dated October 24, 1991. The six reports are as follows:

Golder Associates, Inc., Firewater Storage and Pump System. Geotechnical Investigation for Westinghouse Idaho Nuclear Company, April 18, 1990. (Reference 2)

Northern Engineering Testing, Inc., Report of Geotechnical Investigation. SIS Geotechnical Evaluation. Idaho Chemical Processing Plant, February 1987. (Reference 3)

EG&G Idaho, Inc., Report of the Geotechnical Investigation for the 7th Bin Set at the Chemical Processing Plant. INEL, August 1984. (Reference 4)

Dames & Moore, Soils and Foundation Investigation. Proposed New Waste Calcining Facility. Idaho National Engineering Laboratory (revised), April 16, 1976. (Reference 5)

Dames & Moore, Report of Foundation Investigation. Flourinal and Fuel Storage Facilities. Chemical Processing Plant. Idaho Falls. Idaho, August 9, 1977. (Reference 6)

EG&G-Idaho, Report of the Geotechnical Investigation for the Fuel Processing Restoration Project at the Chemical Processing Plant. INEL, February 3, 1984. (Reference 7)

3. EVALUATION

The six sites identified in Section 2 surround the HLWTFR Project site and are within a distance of 1,400 feet.

All of the reports contain information which can be used to establish the general soil conditions. The data indicates that the sites are very similar. On a macro scale the site conditions are found to be very uniform. Each referenced report has identified a subsurface profile consisting of an alluvial formation of sands and gravel with some silt underlain by an old alluvium consisting predominantly of fine sands and silts. The alluvial formations are underlain by the basalt bedrock. Because of this uniformity on a macro scale, the reference report's distance from the HLWTFR Project site is irrelevant for evaluating the probable site characteristics.

The ground surface is essentially flat. Throughout the entire area covered by the six site investigations, the surface elevation varies within a narrow range between 4,910 feet and 4,913 feet, except for a single boring log reporting the surface at elevation 4,904 feet.

3.1 DISTANCE TO HLWTFR PROJECT

To estimate the distance of each reference project, a center of project location was estimated for each and for the HLWTFR Project. The distance listed in Table 1 is the approximate distance between the estimated center of the HLWTFR Project and the other estimated centers. Figure 1 shows the various site locations.

3.2 MATERIAL CLASSIFICATION

The reports recognize three major units:

- **Alluvium** - An erratic deposit of sand/gravel mixtures. On a large scale, this deposit is dense and uniform and is typically assigned the classification symbol SW-GW (well graded sand - well graded gravel). On a small scale, within a few feet, the density and composition of the mix varies dramatically and lenses of pure sand or gravel can be found. This is normal for alluvial deposits and is confirmed by the wide variation of blow counts reported in the boring logs. For the most part the material is quite permeable. The deposit thickness ranges from 35 feet to 45 feet and may or may not be underlain by old alluvium.

- **Old Alluvium** - A silty and clayey material with the classification symbol ML (silts and very fine sand). Several of the reports refer to this material as "loess," wind-blown silt. In the six referenced reports, 50 borings reported encountering rock, and 27 reported the existence of old alluvium overlaying the rock. The old alluvium ranged in thickness from 0.8 to 17.5 feet. It was noted that old alluvium tended to be present where the relative rock elevation was depressed, and to be absent at locations of relatively high rock elevations. A similar partial layer of old alluvium over the rock can be expected at the HLWTFR Project site.

The loess has a relatively low permeability and may prevent rapid drainage of rain or flood water into the more permeable basalt bedrock.

- Basalt Bedrock - A hard volcanic deposit with occasional fissures and voids. This is normal for volcanic deposits. The surface of the bedrock is not as flat as the ground surface and varies from elevation 4,854 feet to 4,879 feet over the area of the six sites. According to Reference 5, Plate 5, the elevation of the rock surface can change as much as 8 feet over a horizontal distance of 60 feet.

3.3 ROCK ELEVATION

The average rock elevation was calculated for each site using the rock elevation calculated from the data given on the boring logs. Figure 1 shows the location of the borings that were drilled to at least rock depth. Reference 6, Boring 12 indicated an unusually deep rock trench and this rock elevation was not used in calculating the average rock elevation for that site. The average rock elevation for each site is listed in Table 2 and the estimated average rock elevation for the HLWTFR Project is shown as the last item.

It is noted that the highest recorded rock elevation was 4,879.6 (Reference 2, Boring 3) and the lowest was 4,854.3 (Reference 6, Boring 12). The largest change in rock elevation encountered between two adjacent borings was 20 feet within a horizontal distance of approximately 150 feet (Reference 6, Borings 11 and 12).

3.4 GROUNDWATER CONDITIONS

The permanent groundwater level is reported to be as deep as 400 to 500 feet below the ground surface (References 5 and 7). However, groundwater was encountered at a depth of 46.5 feet in one boring (Reference 3, Boring BS) and high water contents were observed in many borings in or above the relatively impermeable old alluvium. Thus, it is plausible that a perched groundwater level can be established in the alluvium during or after heavy rains or floods. Reference 3, page 3, states that the probable maximum flood stage is at elevation 4,918 feet, which is 5 to 8 feet above grade. The elevation to be used in the HLWTFR Project for flood elevation will be as per the Project Design Criteria (Reference 11).

4. EXPECTED SOIL PROPERTIES

All of the materials encountered are homogeneous on a large scale and erratic on a small scale. The latter may not be too important in a dynamic sense since the wavelengths of the seismic waves are large compared to the dimensions of the inhomogeneities. We recommend that for analysis purposes each of the layers be assigned uniform dynamic properties or, for the case of the alluvium, properties which vary smoothly with depth as explained under the heading "Shear Modulus (G)" below.

Based upon the information available, we cannot calculate meaningful statistics for all soil properties. This current available information is complemented by the experienced judgment of HLWTFR Project team members. Table 3 lists our best estimates of expected soil properties and corresponding confidence levels.

The confidence levels shown in Table 3 cannot be defined in terms of probability levels. The confidence levels shown represent the range of test values expected from the geotechnical investigation to be performed at a later date (Reference 1). The confidence levels represent the opinion of the HLWTFR Project team members.

4.1 SOIL BEARING CAPACITY

Safe bearing capacity for the alluvial soil is dependent on footing size and footing embedment. Values given in the six referenced reports range from 5 to 8 kips/ft^2 for various footing sizes and embedment depths. Table 3, item 2, shows a value of 6.5 kips/ft^2 with a $\pm$1.5 kips/ft^2 confidence level.

For rock, References 4, 5, and 7 all recommend a value of 25 kips/ft^2 safe-bearing which is the value shown in Table 3. The bearing capacity of engineered fill is, in our opinion, the same as that of native soil as attested by unit weights obtained for engineered fill (Table 3).

Foundation support in the old alluvium is not recommended in any of the references nor is it relevant for the HLWTFR Project. For earthquakes, two references recommend a one-third increase in allowable bearing in the alluvial soil and one reference recommends a 60 percent increase for the operational earthquake. Table 3 shows the more typical value of one-third increase. No recommendations are given in the references for increased bearing capacity in rock for earthquake conditions.

4.2 FRICTION COEFFICIENT

Three references recommend values of sliding friction on the alluvium ranging from 0.45 to 0.75. A value of 0.5 is shown in Table 3. For rock, two references provide a recommendation for sliding friction, both giving a value of 1.0, which is shown in Table 3.

4.3 LATERAL SOIL PRESSURE COEFFICIENTS

Active pressure recommendations are provided by two references for alluvial soil backfill. The equivalent fluid pressure ranges from 30 to 32 lb/ft^3 and one of the two references also gives the active pressure coefficient as 0.23. Table 3 shows 32 lb/ft^3 equivalent fluid pressure and 0.23 as the coefficient.

The at-rest pressure recommendation was provided by three references:

- One recommends both the equivalent fluid pressure and the coefficient
- One recommends only the coefficient
- One recommends only the equivalent fluid pressure

The equivalent fluid pressure values ranged from 52 to 65 lb/ft^3 and the coefficient ranged from 0.375 to 0.38. Table 3 shows 53 lb/ft^3 as the equivalent fluid pressure and 0.38 as the at-rest pressure coefficient.

The passive pressure recommendations were provided by three references:

- One recommends both the equivalent fluid pressure and the coefficient
- One recommends only the coefficient
- One recommends only the equivalent fluid pressure

The equivalent fluid pressure values ranged from 525 to 590 lb/ft^3 and the coefficient ranged from 3.75 to 4.3. Table 3 shows 557 as the equivalent fluid pressure and 4.0 as the passive pressure coefficient.

4.4 TOTAL UNIT WEIGHT (γ)

All of the reports contain measured values of unit weights of some of the materials encountered. Since the measurements are made at small, localized points, the values reported vary significantly within the same unit, which is normal for alluvial and volcanic deposits. Table 3 shows the best estimate for the total unit weights of the different units. The confidence limits shown in the table are estimates for the overall dynamic behavior of the materials, not for the local variation of unit weights within the individual units. The values shown assume no flooding or saturation of the site.

Bulk densities, dry density plus water content, were measured and reported by four references for the alluvium, by three references for the old alluvium, and by five references for the in-situ rock. The values recommended by the references vary from 123 to 133 lb/ft^3 for the alluvium, 125 to 128 for the older alluvium, and 149.6 to 154 lb/ft^3 for the rock. Reference 2 has been omitted because only near-surface rock was used, and Reference 7 gives only a range of 156 to 165 lb/ft^3 without giving an average value. Table 3 shows a bulk density for in-situ materials of 127 lb/ft^3 for the alluvium, 127 lb/ft^3 for the old alluvium, and 150 for the rock. The bulk density of compacted alluvium (backfill) is given by three references and ranges from 130 to 139 lb/ft^3 using 95 percent of the maximum. Table 3 shows 134.5 lb/ft^3.

4.5 MODULUS OF SUBGRADE REACTION

Recommendations for the vertical modulus of the alluvium is given by three references and these values range from 150 to 500 ton/ft^3. Table 3 shows a value of 350 ton/ft^3. No values are reported for the old alluvium, rock, or for the horizontal modulus of subgrade reaction.

4.6 SHEAR WAVE VELOCITY (V_s)

This is the most critical data for dynamic analysis and it is best obtained by cross-hole or down-hole field measurements. Only References 3, 5, and 6 contain data from such tests. Best estimates, shown in Table 3, are based on this data and engineering judgment. Because of the limited amount of available information, best estimate values are estimates, not statistical averages. The estimate gives more weight to Reference 3 since it is the most recent report and is considered more reliable. The velocities shown for the alluvium correspond to a depth of 20 feet.

The references' measured values for shear wave velocity using either cross-hole or down-hole techniques are as follows. For alluvium, the value ranges from 1,000 to 1,400 ft/sec, for the old alluvium from 500 to 1,150 ft/sec, and for the rock, 3,500 to 4,000 ft/sec. Two references give data obtained from other reference sources and these values also fall within the above-listed range limits. Table 3 shows 1,250, 1,100, and 3,800 ft/sec for alluvium, old alluvium, and rock, respectively.

4.7 P-WAVE VELOCITY (V_p)

This data is less critical and also is best obtained by cross-hole or down-hole field measurements. Only References 3, 5, and 6 contain data from such tests and the best estimates, shown in Table 3, are based on this data. The values shown assume no flooding or saturation of the site. Should it become necessary to perform an analysis for a saturated site, a P-wave velocity of about 5,000 ft/sec, the P-wave velocity in water, should be used for the saturated zone(s) of the two alluvial materials. The shear wave velocities need not be adjusted for this condition.

The data for the compression wave velocity was obtained from the same references as described in Section 4.6 for the shear wave velocity. For alluvium, the range of values is from 2,000 to approximately 4,000 ft/sec, for old alluvium from 1,500 to 2,600 ft/sec, and for rock from 6,800 to 7,500 ft/sec. Table 3 shows 3,000, 2,500, and 7,500 ft/sec for alluvium, old alluvium, and rock, respectively.

4.8 DYNAMIC POISSON'S RATIO (ν)

Poisson's ratio is reported by five references with some values measured, some calculated from geophysical data, and some taken from the literature. In some instances, there is considerable scatter in the data. Values reported for alluvium range from 0.32 to 0.43, for old alluvium from 0.38 to 0.39, and for rock from 0.115 to 0.39.

In this report, the Dynamic Poisson's ratio is determined by the ratio between the above-listed velocities through Reference 10.

$$\nu = \frac{1 - 2\,(V_s/V_p)^2}{2 - 2\,(V_s/V_p)^2}$$

Values shown in Table 3 are 0.395, 0.38, and 0.327 for alluvium, old alluvium, and rock, respectively.

4.9 DYNAMIC SHEAR MODULUS (G)

This property is not measured directly. Its low-strain value is determined from the field-measured values of the shear wave velocity, V_s through Reference 10.

$$G_{max} = \frac{\gamma}{g} \cdot (V_s)^2 \quad ; \quad g = 32.18 \text{ ft/sec}^2$$

and values at higher strains are determined by degradation curves (values of G/G_{max} vs shear strain amplitude), the shape of which are determined in the laboratory from cyclic tests on reconstituted samples. Laboratory data for such curves are presented in Reference 5, Plate 12 and Reference 6, Plate AA. The data is not particularly good, due to sample disturbance, and the authors of both references recommend that the degradation curves of References 8 and 9 be used. We concur with this recommendation. General practice is to use values developed by Seed and Idriss (1970, Soil Moduli and Damping Factors for Dynamic Response Analysis: Report No. EERC 70-10, University of California, Earthquake Engineering Research Center, Berkeley) and Seed, Wong, Idriss, and Tokimatsu (1986, Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils, Journal of the Geotechnical Engineering Division, ASCE, Vol. 12, No. 11, November 1986) as referred to in three references.

The degradation curve for gravel, Reference 9, Figure 17, can be used for the alluvium and the degradation curve for sand, Reference 8, Figure 7, can be used for the old alluvium. The reason for the latter is that we consider the degradation curves for clay in Reference 8 to be obsolete and that the loess of the old alluvium is a low-plasticity material (ML) which behaves like fine sand.

The G_{max}-value provided in Table 3 for the alluvium corresponds to a depth of z=20 feet. The low-strain shear moduli at other depths can be calculated from Reference 8:

$$G_{max} = G_{20} \cdot \sqrt{z[ft]/20}$$

For the purpose of response analyses, the stiffness of the old alluvium and the basalt bedrock can be assumed to be independent of depth, and the stiffness of the basalt can be assumed to be strain-independent.

4.10 DYNAMIC YOUNG'S MODULUS (E)

Four references provide values for Young's modulus. Reference 3 values for alluvium and rock were calculated from shear wave velocity values and are considered more reliable. References 4, 5, and 7 give values which are either static estimates or taken from other references. The range of reported values are as follows: for alluvium, 1,000 to 22,000 kips/ft^2, for rock, 1.9 to 5.9 x 10^5 kips/ft^2.

In this report, this material property is determined by basic theory of elasticity as:

$$E = 2G\,(1 + \nu)$$

Table 3 shows 17,000, 13,000, and 180,000 (kips/ft^2 in rounded figures) for alluvium, old alluvium, and rock, respectively.

4.11 DAMPING RATIO (β)

This quantity is known by structural engineers as the fraction of critical damping. Currently, it cannot be measured in the field and is determined in the laboratory from cyclic tests on reconstituted samples. Laboratory data for such curves are presented in Reference 5, Plate 12 and Reference 6, Plate 4A. As was the case for the G-data, the β-data obtained is not particularly good and the authors of both references recommend that the damping curves of References 8 and 9 be used. We concur with this recommendation.

The damping curve for sand, Reference 8, Figure 10, or Reference 9, Figure 20, can be used for both alluvium and old alluvium.

A constant damping ratio, $\beta = 1\%$, can be used for the basalt. This value has not been measured and is therefore uncertain. However, experience has shown that computed response values are insensitive to the damping value chosen for bedrock.

5. USE OF ESTIMATED VALUES

While individual soil material test values can show significant differences, on a scale the site conditions are found to be very uniform. Actual individual geotechnical survey results (Reference 1) might fall outside the confidence levels noted in Table 3. In our opinion, there will be no need to repeat the calculations for dynamic analyses if the new geotechnical survey best estimates fall within 0.5 to 2.0 of the best estimates in Table 3. Within this range, the response corresponding to the new estimates can be approximated by interpolation between the response values obtained using the initial best estimates. For static analyses, there will be no need to review the calculations if the geotechnical survey best estimates fall within the confidence bounds in Table 3.

5.1 STATIC ANALYSES

For the purpose of static analyses, we recommend using the bearing capacity as a function of foundation depth size and shape from the reference reports. For other parameters we recommend the use of best estimate values from Table 3. Conservative procedures and factors of safety used in static analysis allow for the uncertainties in the best estimate values.

Excavation slopes designed at 1.5 to 1 are suitable for the alluvial soils. Shoring for the vertical portion of the excavation can be achieved with a system of soldier piles and lagging.

5.2 SOIL STRUCTURE INTERACTION DYNAMIC ANALYSES

For the purpose of dynamic analyses, we recommend the use of best estimate values. We recommend that uncertainties in soil properties, defined in terms of shear moduli and soil hysteretic damping ratio, be addressed by using both 0.5 and 2.0 of the best estimate values (Reference 11). Design will be based on envelope values from these three soil profiles.

While it is not possible to guarantee that all the geotechnical test results will fall within the bounds of the analysis, we feel confident that the bounds recommended for the soil structure interaction dynamic analysis are adequate.

5.3 IMPACT OF THE OLD ALLUVIUM LAYER ON DYNAMIC ANALYSES

Half of the test borings available show the existence of an old alluvium layer ranging in thickness from 0.8 to 17.5 feet in the general vicinity of th HLWTFR Project. The effect of this layer of old alluvium on seismic behavior of the HLWTFR Project requires evaluation.

We recommend that analyses be performed using bounding assumptions for the thickness of this potential layer. Further, we recommend that conservative procedures be adopted to account for the effect of this potential layer.

Conservative procedures could be: 1) the use of envelope values, or 2) scaling of results obtained using one specific layer thickness assumption.

APPENDIX A
REFERENCES

REFERENCES

1. *Procurement Specification for Professional Geotechnical Consulting Engineering Services,* High Level Waste Tank Farm Project, ICF Kaiser Engineers, Specification P-10201-X001, Revision 0, dated October 1991.

2. Golder Associates Inc., *Firewater Storage and Pump System, Geotechnical Investigation,* for Westinghouse Idaho Nuclear Company, Idaho Falls Idaho, April 18, 1990.

3. Northern Engineering and Testing, Inc., *Report of Geotechnical Investigation SIS Geotechnical Evaluations,* Idaho Chemical Processing Plants, Idaho National Engineering Laboratory, February 1987.

4. EG&G-Idaho, Inc., *Report of the Geotechnical Investigation for the 7th Bin Set at the Chemical Processing Plants,* Idaho National Engineering Laboratory (INEL), August 1984.

5. Dames and Moore, *Soils/Foundation Investigation,* New Waste Calcining Facility, INEL, Revised, April 16. 1976.

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7. EG&G-Idaho, Inc., *Report of Geotechnical Investigation*, for the Fuel Processing Restoration Project at the Chemical Processing Plant, INEL, February 1984.

8. Seed, H. B. and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Response Analysis," Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley, California, December 1970.

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10. Blevins, Robert, "Formulas for Natural Frequency and Mode Shape," Van Nostrand Reinhold, 1979.

11. Westinhouse Idaho Nuclear Company, Inc., *Project Design Criteria,* for the High Level Waste Tank Farm Replacement Project, Revision 2, February 1991.

APPENDIX B
FIGURES AND TABLES

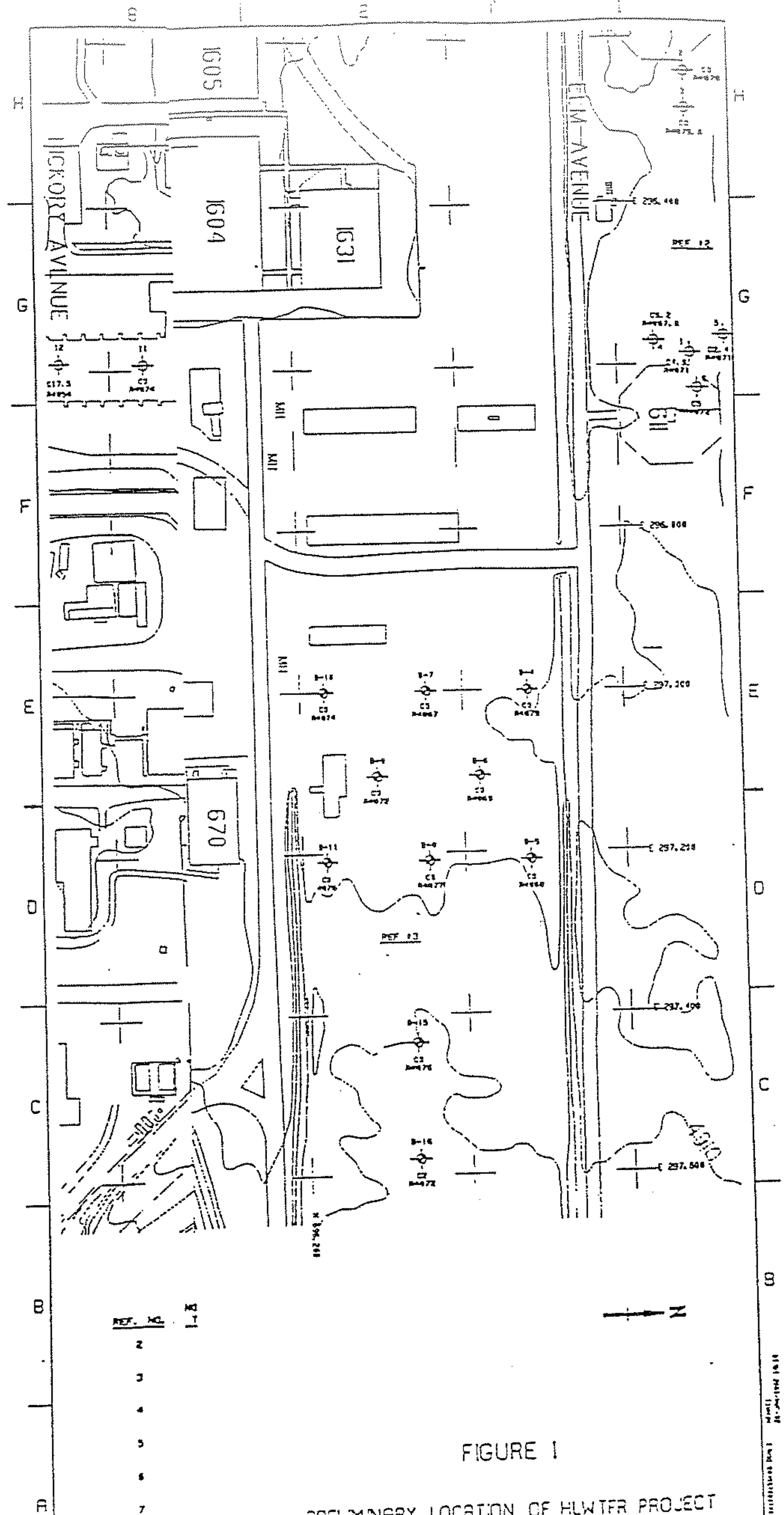

FIGURE 1

PRELIMINARY LOCATION OF HLWTF PROJECT
WITH RESPECT TO REFERENCE SITES

TABLE 1
LOCATION OF SITES IN RELATION TO HLWTFR PROJECT

Ref. No.	Site	Year of Report	Distance from HLWTFR Project	Direction from HLWTFR Project
2.	Firewater Storage and Pump System	1990	1,000 ft	NW
3.	Special Isotope Separation Facility	1987	500 ft	NE
4.	7th Bin Set - Chemical Processing Plant	1984	800 ft	ESE
5.	New Waste Calcining Facility	1976	500 ft	SSE
6.	Flourinel and Fuel Storage Facilities	1977	1,400 ft	SSW
7.	Fuel Processing Restoration Project	1984	1,300 ft	SSE

TABLE 2
ROCK ELEVATION

Ref. No.	Site	Top of Rock Average Elevation
2.	Firewater Storage and Pump System	4,873
3.	Special Isotope Separation Facility	4,873
4.	7th Bin Set - Chemical Processing Plant	4,865
5.	New Waste Calcining Facility	4,869
6.	Flourinel and Fuel Storage Facilities	4,874
7.	Fuel Processing Restoration Project	4,870
NA	HLWTFR Project	4,871 (Best Estimate)

TABLE 3

EXPECTED SOIL PROPERTIES FOR HLWTFR PROJECT

Parameter	Best Estimate	Confidence Level	References
1. Average Rock Elevation (ft)	4,871	±1.5	2 - 7
2. Old Aluvium Layer Thickness (ft)	9	±9	2 - 7
3. Bearing Capacity (kips/ft^2) Alluvium (native soil or engineered fill) increase for Seismic and Wind Rock	 6.5 +1/3 25	 ±1.5 ±0 ±0	 2 - 7 4, 5 & 7 2, 5 & 6
4. Sliding Friction Alluvium Rock	 0.5 1.0	 -0.05, + 0.2 ±0	 3, 6 & 7 6 & 7
5. Lateral Earth Pressure (Alluvium Backfill) Active (Coef./lb/ft^3) At-Rest (Coef./lb/ft^3) Passive (Coef./lb/ft^3)	 0.23/32 0.38/53 4.0/557	 ±0.04/±2 ±0.03/±12 ±0.3/±3.3	 3 & 6 3, 6 & 7 3, 6 & 7
6. Unit Weight (lb/ft^3) In-Situ Alluvium In-Situ Old Alluvium In-Situ Rock Backfill	 127 127 150 134.5	 ±4 ±4 ±5 ±4.5	 2, 3, 5 & 6 2, 5 & 6 2, 4, 5, 6 & 7 3, 4 & 7
7. Modulus of Subgrade Reaction Alluvium (ton/ft^3)	 350	 ±150	 3, 4 & 7
8. Shear Wave Velocity (ft/sec) Alluvium Old Alluvium Rock	 1,250 1,100 3,800	 ±200 ±300 ±500	 3, 4, 5, 6 & 7 3, 4, 5, 6 & 7 3, 4, 5, 6 & 7
9. Compression Wave Velocity (V_p) (ft/sec) Alluvium Old Alluvium Rock	 3,000 2,500 7,500	 ±500 ±400 ±1000	 3, 4, 5, 6 & 7 3, 4, 5, 6 & 7 3, 4, 5, 6 & 7
10. Dynamic Poisson's Ratio (ν) Alluvium Old Alluvium Rock	 0.395 0.38 0.327	 ±0.03 ±0.03 ±0.06	Calculated values
11. Dynamic Shear Modulus (G_{max}) (kips/ft^2) Alluvium (at z = 20 ft) Old Alluvium Basalt	 6,200 4,800 6,700	 ±2,000 ±1,700 ±18,000	Calculated values rounded off
12. Dynamic Young's Modulus (kips/ft^2) Alluvium (at z = 20 ft) Old Alluvium Rock	 17,000 13,000 180,000	 ±5000 ±5000 ±50,000	Calculated values rounded off